DEVELOPMENT OF A REGIONAL HYDROLOGIC MODEL FOR SOUTH FLORIDA

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A complex regional hydrologic systems consisting of thousands of miles of networked canals, sloughs, highly pervious aquifers, open areas subjected to overland flow and sheet flow, agricultural areas and rapidly growing urban areas exists throughout South Florida. This region is faced with equally complex problems related to water supply, flood control and water quality management. Reliance on advanced computational methods and super fast computers alone had limited success with analyzing and solving modern day problems such as these because the challenge is to represent the complexity of the hydrologic system, while maintaining computational efficiency and acceptable levels of numerical errors. Use of object oriented design methods, advanced computational techniques, XML (extensible markup language) and GIS (geographic information system) have become instrumental in the development of a new, physically based hydrologic model for South Florida called the Regional Simulation Model (RSM).

The RSM uses a finite volume (FV) method to simulate 2-D surface and groundwater flow. It is capable of working with unstructured triangular and rectangular mesh discretizations. The discretized control volumes for 2-D flow, canal flow and lake flow are treated as abstract ”water bodies” that are connected by abstract ”water movers”. The numerical procedure is designed to work with these and many other abstractions. An object oriented (OO) code design is used to provide robust and highly extensible software architecture. A weighted implicit numerical method is used to keep the

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model fully integrated and stable. A limited error analysis was carried out and the results were compared with analytical error estimates. The paper describes an application of the model to the L-8 basin in South Florida and the strength of this approach in developing models over complex areas.

INTRODUCTION

South Florida is one of the most complex hydrologic systems in the nation. Its complexity is mainly due to the considerable groundwater and surface-water interaction, spatial variability in land use, hundreds of flow control structures, extensive wetlands systems, adjacent urban areas, influence of Lake Okeechobee, and the unique flow characteristics of the Everglades and the water conservation areas. Even though the computing power has increased every year, the complexity of the system and the management issues in South Florida have increased at an even faster rate. As a result, there is an increasing need to use more efficient computational methods, more flexible computer codes, better code development environments, and better code maintenance procedures to keep pace with these growing demands. The needs for clean code design, participation by multiple developers from a variety of disciplines, and regular use of test cases to check code integrity from time to time have become critical. A number of new technologies have contributed to resolve some of these problems.

The first contribution came from recent developments in information technology and the use of object oriented (OO) code design methods. The use of extensible markup language XML, geographic information system (GIS) technology and database support has allowed for a level of code flexibility and data integration that did not exist earlier. Object oriented methods have been used in the past for hydraulic model design by Solomontine (1996), Tisdale (1996), and many others. Although OO design may have been previously considered to be outside the expertise of many hydrologists, the increased complexity of the hydrologic processes involved, and the need to incorporate work by professionals from many disciplines such as biology, hydrogeology, ecology etc. has changed this attitude. The strong dependencies between hydrology, nutrient transport and ecology have created a need to integrate approaches and therefore to integrate computer codes.

Simple models that address one issue at a time have become inadequate for studying complex systems. The improved use of GIS support tools and the XML language have made it easy to organize and present large amounts of complex data.

The second contribution came from developments in computational methods. Use of unstructured meshes of variable size to simulate 2-D integrated overland and groundwater flow in irregular shaped domains has become common. Full and partial integration with canal networks and lakes is now possible. In the past two decades, a number of physically based, distributed-parameter models have emerged with such features. The early models include MODBRANCH by Swain and Wexler (1997), MODNET by Walten and Wexler (2000), Mike SHE based on Abbott, et al., (1986a and b), WASH123 by Yeh et al. (2000), MODFLOW-HMS by HydroGeoLogic (2000), and models by VanderKwaak (1999), (1998), Schmidt and Roig (1997), and Lal (1989). The computational engines of these models are based on solving a form of the shallow water equations for surface flow and either the variably saturated Richards' Equation or the fully saturated groundwater flow equations. Inertia terms in the shallow water equations were neglected in many of the models and, the models could be solved with a number of the governing equations in the same global matrix. A number of features are available in these models to simulate structures, urban areas and agricultural areas. The choice of features depends on the intended application of the model.

Some developments in numerical error analysis by Hirsch (1989) and Lal (2000) also helped in the selection of optimal discretizations in integrated models. Results of error analysis are useful in developing error indicators that can be used to detect large errors and incipient instabilities of integrated models. Large-scale integration using implicit methods is practically impossible without understanding numerical error and instability.

The third contribution came from a new generation of computer packages that can be used to solve large sparse systems of equations efficiently. It is now possible to develop implicit finite volume algorithms and solve many complex equations simultaneously without iterating between various model components. Modern solvers such as PETSC (Balay, 2001) support parallel processing, and have a variety of built-in tools and options to achieve fast model runs. These solvers are easy to use because details such as matrix storage methods are hidden from the user. The current model uses PETSC to solve the matrices.

The most commonly used integrated model in South Florida is the South Florida Water Management Model (SFWMM) (SFWMD, 1999). It has been adopted to simulate regional hydrology and water management since late 1970's. It simulates the hydrology and the management of the water resources system from Lake Okeechobee in the north to Florida Bay in the south, covering an area of 7600 square miles with a mesh of 1.61 km by 1.61 km (2 mile by 2 mile) cells. The model simulates the major components of the hydrologic cycle including rainfall, evapotranspiration, overland and groundwater flow, canal flow, canal seepage, levee seepage and groundwater pumping. It incorporates current or proposed water management protocols and operational rules. The ability of the model to simulate water management practices and policies that affect urban, agricultural, and environmental water uses in South Florida is one of its major strengths. The success of the model has resulted in an increase in the demand for its use and the growth of its size and complexity beyond what was originally intended. The code gradually became very complex, difficult to understand, improve, and expand. This was a primary factor that contributed to the launching of a new generation regional simulation model (RSM). Unlike the SFWMM that was written in FORTRAN, the RSM code is being developed using an Object Oriented (OO) design and the C++ language. These choices were made so that the code design can allow for easy modification, growth, and multiple developers.

The RSM is functionally a combination of a hydrologic simulation engine (HSE), which executes the numerical flow simulations, and a management simulation engine (MSE) that can represent structure and pump operations. The HSE has been used to simulate flow in the Kissimmee River by Lal, (1998), and in Everglades National Park by Lal, et al., (1998), and Brion, et al. (2000 and 2001). The accuracy of the model was verified using the MODFLOW model and an analytical solution for stream-aquifer interaction (Lal, 2001). This paper describes the object oriented design of the HSE that makes it possible to fully integrate the components of the system, and allow for expansion using new hydraulic components, land use types, micro-hydrological features, canal section types and other aspects of the system.

Numerical models can give inaccurate results if not used properly. Results of numerical error analysis can be used to obtain ranges of spatial and temporal discretizations that are suitable for a model to limit numerical errors. A study was carried out to determine the relationship of the numerical error to the size of the triangular cells and length of the time step. Results of this study are included in the paper so that they can be useful in the design of discretizations.

The paper also describes an example of the application of this model to simulate hydrology in the L-8 basin of South Florida. The basin is delineated by artificial levees, and consists of natural, agricultural and urban areas adjacent to each other. A canal network partially overlaps the basin and a system of levees blocks flow at various sections. A number of structures are used to pass water between various overland flow cells and canal segments. The L-8 basin is a relatively simple basin in South Florida, but represents some of the complexities of the regional hydrologic system. Many of these complexities apply to conditions outside South Florida as well. The results of the simulation are used to demonstrate why this approach was chosen to develop the new regional simulation model (RSM) for South Florida.

GOVERNING EQUATIONS

The governing equations for the integrated overland - groundwater - canal - lake flow system consist of mass balance equations and equations of motion. In many overland flow systems, the equation of motion can be reduced to a friction flow type equation by neglecting inertia terms. Governing equations in conservative form are used in the current implicit implementation of the finite volume method.

Overland flow

After neglecting the inertia terms, the 2-D Saint Venant equations, which consist of a continuity equation and an equation of motion, reduce to a simpler form. The continuity equation for both overland flow and saturated groundwater flow in the regional system becomes

$$s_c \frac{\partial H}{\partial t} + \frac{\partial (Hu)}{\partial x} + \frac{\partial (Hv)}{\partial y} - R_{rchg} + W = 0 \tag{1}$$

in which, u and v are the velocities in the x and y directions; H = water head; R_{rchg} = net contribution of recharge from local hydrology into the regional system; W = source or sink terms due to pumping wells, etc.; and s_c = storage coefficient. $s_c = 1$ for overland flow. The term R_{rchg} is computed for each of the discretized cells of the distributed system, assuming that a pseudo cell exists for each cell. Each pseudo cell is designed to capture all of the complexities related to local hydrology. A number of pseudo cell models are described later.

For overland flow under diffusion flow conditions (Akan and Yen, 1981) and for groundwater flow, average flow velocities in x and y directions are defined as

$$u = -\frac{T}{d}\frac{\partial H}{\partial x}, \quad v = -\frac{T}{d}\frac{\partial H}{\partial y} \tag{2}$$

For groundwater flow, $T = T(H)$ = transmissivity of the aquifer as a function of the water level; d = water depth. For overland flow, $T = C(H)S^{\lambda-1}$ in which $C(H)$ is defined as the conveyance; S = water surface slope and λ = an empirical constant that is described later. Both $T(H)$ and $C(H)$ are useful in describing a whole range of overland and groundwater flow behaviors. In the model, object oriented design methods allow for the implementation of a variety of options for the functions $T(H)$ and $C(H)$ by making them as base class objects with metamorphic behavior. These objects can express the behavior of constants, analytic functions or lookup tables based on field experiments. This type of abstract representation of $T(H)$ and $C(H)$ is useful in the OO design in describing specialized flows as in the case of ridge and slough flow of wetlands.

Manning's equation is commonly used to describe overland and canal flow. When using a general form of the Manning's equation described as $V = (1/n_b)d^{\gamma}S^{\lambda}$, the corresponding expression for T can be expressed as

$$T(H) = \frac{d^{\gamma+1}S^{\lambda-1}}{n_b} \tag{3}$$

For the commonly used form of Manning's equation, $\gamma = 2/3$; $\lambda = 1/2$; V = flow velocity; S = water surface slope and n_b = Manning coefficient. The Manning's equation can be used to describe the discharge per unit width as $q(H) = T(H)S = C(H)S^{\lambda}$. Functions $T(H)$ and $C(H)$ can be described using a variety of methods such as functions and lookup tables.

Because of the nonlinear nature of the Mannings equation and its singularity at $S = 0$, linearization can be a problem at small slopes. To avoid the resulting floating point overflow, $S = Max(S, \delta_n)$ is used when $\lambda < 1$. The variable δ_n was originally used as a numerical adjustment parameter, but it can also be used to separate turbulent flow from laminar flow. A large value of δ_n would result in a small value of T and makes the matrix computations more accurate, but it would also force larger areas of relatively flat landscapes to have laminar-like flow. A value of $\delta_n = 10^{-13} - 10^{-7}$ is used in the flat terrain of South Florida. Equation 3 can also be used in wetlands by selecting the parameters suggested by Kadlec and Knight (1996).

Canal fbw

The 1-D St Venant equations are used to describe canal flow. The continuity equation in conservative form is

$$\frac{\partial A_c}{\partial t} + \frac{\partial Q}{\partial n} - R_{canal} + W = 0 \tag{4}$$

in which, A_c = cross sectional area of the canal; Q = discharge rate; n = distance along the canal; R_{canal} = rate (m^2/s) at which water is entering the canal due to seepage and other sources per unit length; W = source and sink terms due to pumps. For canals under diffusion flow conditions, $Q = C(R)\sqrt{S} = AR^{2/3}\sqrt{S}/n_b$. This can be linearized as

$$Q = T_c \frac{\partial h}{\partial n} \tag{5}$$

in which, h = canal water level; T_c is defined as $AR^{(2/3)}/(n_b\sqrt{S})$ using the Mannings equation; $C(R)$ = canal conveyance; R = hydraulic radius. Energy slope S is defined as $Max(S, \delta_n)$ with $\delta_n = 10^{-13} - 10^{-7}$ to avoid the singularity at $S0$.

Lake fbw

The equation governing mass balance in a lake is

$$A_l \frac{\partial H_l}{\partial t} - R_{lake} + W = 0 \tag{6}$$

in which, A_l = lake area; H_l = water level; R_{lake} = net volume rate at which water is entering the lake water body due to leakage. The governing equations written in conservative form are used in the implicit implementation of the finite volume method.

Recharge from the local hydrologic system

The local hydrology in a regional system can depend on the local land use type. Different land use types generate different recharges and therefore different hydrologic responses. The recharge R_{rchg} described in (1) therefore has to be computed separately for each cell covering the model. The computations take into account ET, rainfall, soil moisture effects, urban detention, local drainage effects and other factors depending on the land use type. The equation of mass balance for the local hydrology in a cell is

$$R_{rchg} = P - E + I - \frac{dU_s}{dt} - \frac{dD}{dt} \tag{7}$$

in which, R_{rchg} = recharge rate (m/s) computed as a volume rate per unit cell area entering into the cell; P = precipitation rate; E = evapotranspiration rate; I = water entering the cell for irrigation and other similar functions; U_s = unsaturated moisture depth; D = detention volume converted to depth. The rates $\frac{dU_s}{dt}$ and $\frac{dD}{dt}$, depend on infiltration and percolation rates of the local cell. In the model, these complex computations are carried within the pseudo cell of each respective cell. Pseudeo cell types are developed for various land use types and permitting conditions. More information about pseudo cells is provided under the object design.

THE IMPLICIT FINITE VOLUME METHOD

Governing equations for overland flow, groundwater flow, canal flow, lake flow and other types of flow are based on conservation laws and can be solved using the finite volume method. The equations can be written in conservative form as

$$\frac{\partial U}{\partial t} + \frac{\partial F}{\partial x} + \frac{\partial G}{\partial y} + S = 0 \tag{8}$$

in which, U is a conservative variable representing H, h, or H_l; variables F and G are the x and y components of flux in 2-D flow and $S = -R_{rchg} + W$ = summation of source/sink terms. The finite volume formulation is applied to all 2-D, 1-D and lake related regional flows. The numerical model is developed for generic control volumes and is applied to all water bodies on an equal basis.

The finite volume formulation governing equation (8) is derived by integrating it over an arbitrary control volume or water body Ω.

$$\frac{\partial}{\partial t}\int_{\Omega} U \, d\Omega + \int_{S} (\mathbf{E}.\mathbf{n}) \, dA + \int_{\Omega} S \, d\Omega = 0 \tag{9}$$

in which, $\mathbf{E} = [F,G]^T$ = flux rate across the wall and $\mathbf{n}$ = unit normal to the wall. The first term of (9) represents the rate of change of water mass in the water body. The second term is obtained using the Gauss' theorem and contains the sum of fluxes crossing the control surfaces of the water bodies. This term contains all flow exchanges among the water bodies. Any mechanism that is capable of moving water between any two water bodies is defined as a water mover. Water bodies and water movers are two of the basic building blocks of the model. They eventually become abstractions in the OO design. These abstractions are capable of growing and evolving into model objects as the model evolves.

In the model, equation (9) is solved for average water heads H, h or H_l of all the water bodies simultaneously. In finite volume formulations, (9) represents a system of differential equations.

$$\frac{d\mathbf{V}}{dt} = \Delta\mathbf{A}(\mathbf{H})\frac{d\mathbf{H}}{dt} = \mathbf{Q}(\mathbf{H}) + \mathbf{S} \tag{10}$$

in which, $\mathbf{H}$ = a vector containing the water heads of all 2-D cells, canal segments, and lakes; $\mathbf{V}$ = volumes of water contained in the water bodies; $\Delta\mathbf{A}$ is a diagonal matrix whose elements (i,i) are defined as the slopes of the stage-volume (SV) relationships of water bodies $i = 1,2,\ldots$. The stage-volume relationship is defined as

$$V_i = f_{sv}(H_i) \tag{11}$$

in which each function $f_{sv}(H_i)$ is a single valued, stage-volume relationships that is defined for the water body i. The inverse relationship is defined as $H_i = f_{vs}(V_i)$. Element i of $\Delta\mathbf{A}(\mathbf{H})$ in (10) is defined as

$$\Delta A_i(H_i) = \frac{\partial f_{sv}(H_i)}{\partial H_i} \tag{12}$$

in which the function $\Delta A_i(H_i)$ is the effective area of water body i. For 2-D open water flow, this is the cell area. For groundwater, this is s_c times the cell area. The third term $\mathbf{Q}(\mathbf{H})$ of (10) gives the summation of all the inflows into each water body due to the action of all the water movers. In order to solve the coupled system, the water mover equations are first linearized to obtain

$$Q_r(\mathbf{H}) = k_0 + k_i H_i + k_j H_j \tag{13}$$

in which $Q_r(\mathbf{H})$ = discharge rate in the water mover r. The water mover r moves water from water body i to water body j. Linearization is performed using partial differentiation or approximate methods. Values k_0, k_i and k_j are applied to build the resistance matrix that is used to linearize $\mathbf{Q}(\mathbf{H})$ as $\mathbf{Q}(\mathbf{H}) = \mathbf{M}.\mathbf{H}$. The ordinary differential equations (10) with linearized $\mathbf{Q}(\mathbf{H})$ are solved using a weighted implicit method. Lal (1998) used the following system of equations to solve (10).

$$[\Delta\mathbf{A} - \alpha\Delta t\mathbf{M}^{n+1}].\Delta\mathbf{H} = \Delta t[\mathbf{M}^n].\mathbf{H}^n + \Delta t[\alpha\mathbf{S}^{n+1} + (1-\alpha)\mathbf{S}^n] \tag{14}$$

in which, α = time weighting factor, assumed to be in the range 0.6-0.8 for small models and close to 1.0 for large models that may otherwise show signs of instability. This equation takes into account water balance of all the water bodies between times t^n and t^{n+1}. Knowing the volumes of water $\mathbf{V}(\mathbf{H^n}) = \mathbf{f_{sv}}(\mathbf{H^n})$ at time step t^n and ΔH, it is possible to compute $\mathbf{V}^{n+1}$ using

$$\mathbf{V^{n+1}} = \mathbf{V^n} + \Delta\mathbf{A}.\Delta\mathbf{H} \tag{15}$$

The new heads $\mathbf{H^{n+1}}$ at time step $n+1$ can now be computed using the storage-volume relationship $\mathbf{H^{n+1}} = \mathbf{f_{vs}}(\mathbf{V^{n+1}})$. The volume to stage conversion is necessary only to compute the heads used to compute the hydraulic driving forces in the water movers. The water balance computation itself does not involve any water heads.

Equations (10)-(15) apply to all the water bodies equally, and the set of equations (14) solves the entire system simultaneously. Water heads $\bar{\mathbf{H}}$ at time $t^n + \alpha\Delta t$ act as the forcing functions for flow during the time step. Values of $\bar{\mathbf{H}}$ are computed as $\bar{\mathbf{H}} = \mathbf{H}^n + \alpha\Delta\mathbf{H}$ and are used to compute the volumes of water that pass between the water bodies in a model. These values add up to $\mathbf{Q}(\bar{\mathbf{H}})\Delta t$ for the water bodies. The water mover discharges $\mathbf{Q_r}(\bar{\mathbf{H}})$ that contribute to equation (14) are computed using (13). The water balance in any water body i can be verified by comparing the change in water volume $\Delta A_i(H_i^{n+1} - H_i^n)$ with the summation of water mover discharges

$Q_r(\bar{H})$.

THE OBJECT DESIGN

The process of abstraction and determining the relationship between abstractions form the basis for OO design. The basic abstractions used in the RSM model include: (i) ”water bodies” that represent discretized cell elements, canal segments and lakes which store water; (ii) ”water movers” that represent the only mechanisms to move water between water bodies; (iii) ”stage-volume relationship functions” (SV) that map between the stages and the volumes in water bodies and (iv) ”pseudo cells” that capture the local hydrology in the water bodies and compute their recharge. The entire hydrologic system can be decomposed into these and other abstract types such as transmissivity and conveyance functions $T(H)$ and $C(H)$. Figure 1 shows some of the basic building blocks of the model. These abstractions allow a single numerical scheme to be used for governing equations describing all flow types.

Abstract data types or classes in the model can be related to other classes through inheritance. A ”subclass” or a ”derived class” of a water body base class for example, can be a discretized overland flow cell, canal segment or a lake. They all inherit properties of the base class. Inheritance makes it possible to use polymorphism in OO modeling and allows functions to behave correctly depending on the object type. Polymorphism also allows water bodies to transform into canals, cells and lakes while water movers can transform into canal flow, overland flow and structure flow. Special methods associated within these objects fill the proper elements in the matrix. Four of the abstract classes used in the model are described below.

Water bodies

The water body is the basic abstraction that collects water conservatively and provides its volume when requested. Water body objects represent control volumes of the finite volume method, and provide a protected status for conservative variables such as water mass and solute mass. Cell elements, canal segments and lakes become polymorphic water bodies. The head of the water body when needed is computed by calling the stage-volume relationship function of the water body as described after (11) and in the following sections. The first terms of (9) and (10) represent water bodies. Figures 1 and 2 show examples of water bodies. They have a polymorphic behavior and change into cells, canal segments, lakes or any other objects depending on the situation. Figure 3 shows part of a class diagram for water bodies which was written using the convention of Rumbaugh, et al. (1991).

Water movers

The water movers are the basic abstractions needed to transfer water between any two complex water bodies. They represent the flux term in the finite volume method, and carry flow across control surfaces as in canal flow, overland flow and all other kinds of flow such as structure flow. They conserve mass and track the progress of water movement. Some water movers such as those for overland flow, groundwater flow and canal flow are created by default, depending on the geometric placement of water bodies. Many more water movers such as structure water movers are added subsequently based on input data. Additional water movers can be written as needed to accommodate new flow types. All water mover objects are placed in object containers to be accessed easily using the C++ Standard Template Library (STL) features (Stroustrup, 2000). Figures 1 and 2 show sketches of sample water movers. Figure 3 shows part of a class diagram for water movers.

Since only water movers can move water between water bodies, the finite volume method provides an easy way to maintain the mass balance of the system. Water movers can operate under gravity or by use of structures and pumps. Some of the movers are described below.

Overland flow water mover

When the water levels are above ground in adjacent cells, overland flow takes place. The discharge in the water mover Q_r as described in (13) is computed using the circumcenter method derived for mixed finite elements (Lal, 1998).

$$Q_r = \Delta l \; T_r \frac{H_m - H_n}{\Delta d_{mn}} \begin{cases} \text{for} \quad H_m > H_n \quad \text{and} \quad H_m > z_m \quad \text{and} \quad H_m > z_n \\ \text{or} \quad H_n > H_m \quad \text{and} \quad H_n > z_n \quad \text{and} \quad H_n > z_m \end{cases} \tag{16}$$

in which, $T_r =$ equivalent inter block transmissivity in the overland flow layer, computed under the assumption that transmissivity varies linearly between circumcenters (Goode and Appel 1992, and McDonald and Harbough 1988), in the MODFLOW model; $d_{mn} =$ distance between circumcenters of triangles m and n and Δl = length of the wall; z_m, z_n = ground elevations of cells m and n. T_r is computed using

$$T_r = \frac{T_m + T_n}{2} \quad \text{for} \quad 0.995 \leq \frac{T_m}{T_n} \leq 1.005 \tag{17}$$

$$T_r = \frac{T_m - T_n}{\ln \frac{T_m}{T_n}} \quad \text{otherwise} \tag{18}$$

T_m and T_n are the values for the cells defined in (2) for overland flow. These values can be computed using methods that were described earlier. Matrix elements filled up by the overland flow water movers are described in the paper by Lal (1998).

Groundwater flow water mover

When simulating groundwater flow, transmissivities are assumed constant within a material in the cell, and the discharge in the water mover Q_r is computed using

$$Q_r = \Delta l \, \frac{H_m - H_n}{(\frac{l_m}{T_m} + \frac{l_n}{T_n})} \tag{19}$$

in which, l_m and l_n are the distances from the circumcenters to the wall; T_m and T_n = transmissivities described in (2).

Canal flow water mover

A canal flow water mover is used when simulating canal flow. A linearly varying conveyance is assumed between canal segments. The equation for discharge between two segments m and n is the same as (16). The value of T_m for segment m is

$$T_m = \frac{A_m}{l_m \sqrt{S_n} n_b} \left(\frac{A_m}{P_m} \right)^{\frac{5}{3}} \tag{20}$$

in which, A_m = average canal cross sectional area of segment m; P_m = average wetted perimeter; n_b = average Manning's roughness and l_m = length of canal segment. When simulating canal networks, each pair of segments of a canal joint is considered as a canal water mover. A canal joint with n limbs has $n(n-1)/2$ canal water movers as a result. All these movers have to be considered to populate the matrix and. Their summation computes the actual discharge.

Canal seepage water mover

The rate of leakage between a canal segment and a cell is described using a canal seepage water mover. The seepage rate q_l per unit length of the canal is given by Darcy's equation.

$$q_l = k_m p \frac{\Delta H}{\delta} \tag{21}$$

in which, k_m = sediment layer conductivity; p = perimeter of the canal subjected to seepage; δ = sediment thickness and ΔH = head drop across the sediment layer.

Structure flow water mover

Linearization of structure equations to fit to the format of (13) is not always easy for most of the structures. Considering the generic form $Q_s = Q_s(H_u, H_d, G)$ for structures in which H_u and H_d = upstream and downstream water levels and G = gate opening, it is possible to carry out partial differentiation of (13) for a limited number of structures. When the function is complicated, linearization is carried out using the two previous calls to function $Q(H_u, H_d, G)$ as shown by Lal (1998). An alternative approximate approach for many low head structures in Florida is to use

$$Q(H_u, H_d) = \frac{Q(H_u^n, H_d^n, G^n)}{H_u^n - H_d^n}(H_u - H_d) \tag{22}$$

in which the superscript n represents values from previous time step.

Lookup table water mover

Lookup table water movers are useful in a number of situations. Both one dimensional (1-D) and two dimensional (2-D) lookup tables can be used to simulate both active and passive flows. Structure flow is an example of passive flow and pump flow is an example of active flow. The relationship used for water movers based on 1-D lookup tables is $Q_s = Q_s(X)$ in which X = water level of any water body or the water level difference between two adjacent water bodies. For water movers based on 2-D lookup tables, $Q_s = Q_s(H_u, H_d)$. Examples of 1-D water movers and their water budgets are shown in the L-8 application.

Stage-volume (SV) relationships

Stage-volume relationships or functions make it possible to use actual surface storage characteristics in the local topography. This feature is useful when the local topography is complex as in the case of ridge-and-slough formations in the Everglades. In the case of cells, SV relationships provide water levels when the water volume is given or vice versa. In the case of canals, they are used to obtain water levels for a variety of commonly used cross section types. Since computations in the model are based on volumes and volume rates in the water bodies and water movers respectively, the relationship $H = f_{vs}(V)$ is used to obtain the water level in a water body, or the water head in confined groundwater flow. The water level is needed because it drives flows in water movers. The relationship $V = f_{sv}(H)$ gives the volume for a given stage, and is used at $t = 0$ to get the initial water volume. Local topography, storage coefficient, canal cross sectional area and other information are used in the SV functions. Stage-volume relationships can be complex functions or lookup tables based on experimental data. Some simple examples of $f_{sv}(H)$ are described below.

SV relationship function $f_{sv}(H)$ for a cell with flat ground

When the ground level is flat, the SV relationship for a single layered aquifer is given by

$$V = f_{sv}(H) = As_c(H - z_b) \quad \text{for} \quad H < z \tag{23}$$

$$V = f_{sv}(H) = As_c(z - z_b) + (H - z) \quad \text{for} \quad H \geq z \tag{24}$$

in which, $z_b =$ elevation at the bottom of the aquifer; $z =$ elevation of the ground and $A =$ cell area.

Inverse relationship $f_{vs}(V)$ for cells with flat bottom

Since the expression for flat ground is linear, the following relationship is obtained.

$$H = f_{vs}(V) = z + \left(\frac{V}{A} - s_c(z - z_b)\right) \quad \text{for} \quad V > As_c(z - z_b) \tag{25}$$

$$H = f_{vs}(V) = z_b \quad \text{for} \quad V < 0 \tag{26}$$

$$H = f_{vs}(V) = z + \frac{V}{A\, s_c} \quad \text{otherwise} \tag{27}$$

$$\tag{28}$$

SV relationship function $f_{sv}(H)$ for a canal segment with a rectangular section

For rectangular canals, the relationship is

$$V = f_{sv}(H) = 0 \quad \text{for} \quad H < z_c \tag{29}$$

$$V = f_{sv}(H) = BL(H - z_c) \quad \text{for} \quad H \geq z_c \tag{30}$$

in which, z_c = elevation of canal bottom; L = length of canal segment and B = canal width. The inverse relationships of most of the functions are complex and are not described here due to their complexity. However it is important to keep them simple and monotonic. When simulating confined flows in layered aquifers, the stage-volume relationship becomes a head - mass relationship because only the mass is actually conserved.

Pseudo cells

The hydrologic system of South Florida covers areas with many types of land uses. Most areas along the east coast are heavily urbanized, while some of the areas in the south are natural areas and wetlands. Areas south of Lake Okeechobee are mostly agricultural. Pseudo cells simulate urban, agricultural, overland flow or any other local sub system. They are used to separate the complexities of the local hydrology from the regional system. Functionally, they are used to compute the contribution of recharge to the regional system. Equation (7) represents the mass balance conditions that determine the recharge in a pseudo cell. It allows for a number of storage mechanisms and a number of complex interchange mechanisms that can occur within the cell, isolated from the regional system. Infiltration, percolation, seepage, and urban drainage are some of the mechanisms available for moving water between different storage components in the pseudo cell.

Equation (7) can be applied for many lumped parameter hydrologic models. As a result, some of the lumped parameter models can be used as pseudo cells of the RSM, with some reorganization. The AFSIRS model (Smajstria 1990) and the CASCADE model (SFWMD, 2001) are two such models that can be used in RSM as agricultural and urban pseudo cells respectively. When necessary, the 1-D Richards equation models or other numerical models can also be used as a pseudo cells. Some of the simple pseudo cells available are described below.

Open water pseudo cells

These are the simplest form of pseudo cells because there is no unsaturated or urban storage within them. The equation for recharge (7) is given simply as

$$R = P - E \tag{31}$$

because there is no unsaturated or detention storage.

Urban area pseudo cells

A lumped parameter pseudo cell model is available as an option for urban cells. With this option, the runoff and the detention are computed using the SCS curve number method. Routing is implemented using a linear reservoir to give the proper time of concentration. The recharge *R* is passed to the regional model as a source and the urban runoff is routed to a designated water body. An example of a ”CASCADE” type urban cell data set, written in XML format, is shown below.

```
<pseudocell
   route = "7"
   tc = "3600"
   kveg = "1.0"
   unsat_deep_depth = "2.0"
   unsat_shal_depth = "0.5"
   fld_cap="0.4">
<\pseudocell>
```

In the data set defined within the `<pseudocell>` tags, the identifier of the water body to which water is routed is 7; the time of concentration `tc` is 3600 s; the ET crop coefficient is 1.0; deep root zone depth = 2.0 m; shallow root zone depth = 0.5 m and field capacity = 0.4. The example here demonstrate both pseudo cells and the XML data format used to input data to the model. The design allows for users to write their own pseudo cells according to the specific local needs.

Boundary conditions

When solving for 1-D and 2-D diffusion flow, only one boundary condition of discharge type or water level type is needed. With diffusion equations, general head, uniform flow, and lookup table boundary conditions are also very useful. The basic 2-D boundary condition types are described in the paper by Lal (1998). Some boundary condition types can be defined for generic water bodies. A number of cell and wall boundary conditions are described below.

The flow boundary condition to a water body is fairly simple to implement because it introduces a known flow rate to a specified water body.

$$Q_i(t) = Q_B(t) \tag{32}$$

in which, $Q_i(t)$ = inflow rate to water body i and $Q_B(t)$ = specified inflow rate. This condition only modifies the right hand side of the matrix.

The head boundary condition states that the water head of a water body is tied to a known constant value or a time series value. When applied to a water body i,

$$H_i(t) = H_B(t) \tag{33}$$

in which $H_i(t)$ = the head at water body i at time t and $H_B(t)$ = assigned value. When used in the implicit solution, the corresponding matrix row is set to zero except for the diagonal term, eliminating the water mover contributions.

A better method is to apply the head boundary condition at cell walls or canal joints instead of water bodies. In order to do this, a discharge q_i is added to the cell to create the effect of having a head value of $H_B(t)$ at the wall. The discharge added is

$$q_i = \frac{T(H)l}{l_c}(H_B(t) - H_i) \tag{34}$$

in which, $T(H)$ = transmissivity; H_i = cell head; l = wall length and l_c = distance from the wall to the circumcenter. Describing the discharge added to the cell, the wall head boundary condition can be satisfied.

The general head boundary condition is described using

$$q_c = K_G\, l(H_B - H_w) \tag{35}$$

in which, K_G = specified conductance value in m/s and H_w = wall boundary head. Values of K_G and H_B are used to specify the BC. The uniform flow boundary condition is similar and relates the uniform slope to a flow rate as follows:

$$q_i = T(H)S_b \tag{36}$$

in which S_b = slope of uniform flow associated with the water body.

Operation of structures and pumps

Some structures and pumps in the model are operated to achieve certain performance goals in the hydrologic system. The operations are based on rules assigned by water managers. Some of the operations are manual and others are automatic. Some operational rules are very complex because they evolved over time depending on historic events, human needs, and prior experiences. Sometimes the complex operational rules and logical directives applied to the structures and pumps are implemented in the model through modification of the status of these facilities. These modifications can be logical on/off type or proportional type.

The purpose of operating structures and pumps is to achieve a desired performance in the system. Some performances and conditions are mandated by legislation. Some of the performance measures used for South Florida are described in a comprehensive review study report (USACE, 1999). In the model, water management is carried out by the operation of structures and pumps to achieve desired goals. Currently the operations are mostly rule-based, and not necessarily optimal. Optimization for higher performances in the future involves determination of the decision variables in the water movers to achieve optimal system performances. The methods available include optimal control methods (Gelb, 1974), linear programming (Louks, et al., 1981), simple rules based on past experiences, and many others. The methods currently used are described in the South Florida Water Management Model documentation (SFWMD, 2001). In the RSM, these methods will operate within part of the computer code referred to as the Management Simulation Engine (MSE). Some simple rules that are used to operate the MSE are described later as part of the L-8 basin example.

Water budgets

In a compartmentalized landscape such as South Florida, determination of the water budget within a hydrologic basin or a compartment can be very important for many applications. Water bodies and water movers are ideally suited to carry out water budget computations because they track the flow of water in the model. Water bodies give the mass contained in them and water movers give the fluxes across their walls. Each water body has an attached list of water movers that can report the discharges. The discharge computation for the water movers is carried out using (13) and the updated values of head and k. An example showing the water budget capabilities of the model is presented in the applications section. Table 1 shows the lists of water movers for the two water bodies in the example shown.

Model errors

Although mass balance errors in the finite volume method are small, the computational errors can be large depending on the discretization. Improper selection of spatial and temporal discretizations can make a model implementation ineffective in producing solutions of a certain scale. Recent studies show that proper discretization can be based on simple rules derived using analytical equations for numerical error (Lal 2000).

Error behavior of triangular cells is similar to the error behavior of triangular cells. In the latter case, the dimensionless spatial discretization $\phi = k\Delta x$ is computed as $\phi = k\sqrt{\Delta A}$ in which $k =$ wave number of the disturbance and $\Delta A =$ cell area. In order to study this error behavior, a confined groundwater model is defined in a 10 $km \times 10$ km square domain with a triangular mesh of 3200 approximately isometric triangles. A 1-D head disturbance is introduced into the domain by changing the head of one of the boundary walls in a sinusoidal manner. Water heads at different distances away from the wall are then monitored over long periods. These heads are compared with their analytical estimate to obtain the measured errors in the amplitude. These errors in amplitude can now be compared with analytical error estimates (Lal 2000).

The analytical solution for the head in 1-D groundwater flow is

$$H(x,t) = H_o e^{-kx} \sin(ft - kx) \tag{37}$$

in which, $f =$ frequency of the boundary disturbance in radians per second; $H_o =$ amplitude of the disturbance; $x =$ distance from the boundary and $k = \sqrt{fs_c/(2T)}$. The analytical solution for the numerical error for this problem is (Lal, 2000).

$$\varepsilon_T(x) = \frac{k\varepsilon}{\beta\phi^2}x = \frac{2k\varepsilon}{\psi}x \tag{38}$$

in which, $\varepsilon_T(x)$ = maximum error as a fraction of the amplitude at a distance x over the duration; Δt = time step; $\psi = f\Delta t$ = dimensionless time step and ε = maximum error per time step as a fraction of the amplitude; $\beta = T\,\Delta t/(s_c\,\Delta A)$. For this problem, it can be shown that $\beta = \psi/(2\phi^2)$. Values of ε can be obtained analytically using an expression given by Lal (2000). Measured values of the same ε are obtained by plotting $\varepsilon_T(x)$ against x for a number of model runs and fitting (38) to determine the slope. Figure 4 shows one such plot when the period is 20 hrs or $f = 8.726 \times 10^{-5}\,s^{-1}$, $\Delta t = 60$ min, $\sqrt{\Delta A} = 152$ m, $T = 2.0m^2/s$ and $s_c = 0.2$ confirming that the error behavior is linear as shown in (38). Using the slope of the graph as 3.0527 $\times 10^{-5}$, $k = \sqrt{fs_c/(2T)} =$ 0.00004264 and $\psi = 0.3146$, the value of ε can be determined using (14) as 0.011246. This compares well with the analytical value 0.011291 obtained by Lal (2000) for $\phi = 0.2049$ and $\psi =$ 0.3142. The model value is shown as a dot in Figure 5 and compares well with the analytical values shown in contours.

The contours of analytical errors in Figure 5 can be used not only to verify the RSM model erors as shown, but also to estimate errors of future model applications. Using (37) and (38), the absolute error ε_{abs} in (m) over the domain during a full cycle can be computed as

$$\varepsilon_{abs} = \frac{2H_0\varepsilon}{e\psi} \tag{39}$$

where $e = 2.718$. This maximum error occurs at a distance $x = 1/k$ from the boundary during the peak and the trough of the cycle.

When using Figure 5 to evaluate a mesh for a future model application, discretizations Δt and ΔA are first used to compute $\psi = f\Delta t$ and $\phi = k\sqrt{\Delta A}$ first. The equation $f = Tk^2/s_c$ is then used to describe the relationship between f and k for groundwater flow. Figure 5 is used to determine ε and (39) is used to determine ε_{abs}. If this error is too large, finer discretizations have to be selected. The second important consideration in selecting a discretization is run time, which is proportional to $k^4/(\psi\phi^2)$ or $f^2/(\psi\phi^2)$ in which k or f describes the level of spatial and temporal detail simulated by the model (Lal, 1998).

MODEL VERIFICATION AND APPLICATIONS

The computational methods used in the RSM model have been used in the past on a number of occasions. The most rigorous verification of the model was carried out using an analytical solution for the problem of stream-aquifer interaction (Lal 2001). The test verified the accuracy of the stream and aquifer solutions along with the stream-aquifer interaction. The FORTRAN version of the model with overland and groundwater capabilities was tested by simulating the Kissimmee River (Lal 1998). The C++ version of the model with 2-D flow capabilities was used to simulate flows in the Everglades National Park by Lal, et al. (1998). The error analysis described earlier is used to verify the accuracy of the model.

The L-8 basin of South Florida was chosen not only to verify the model accuracy but also to demonstrate some features of the RSM model that are designed to handle complexity. Many of the hydrologic components and issues related to the L-8 basin are not simple and typical for South Florida. Although it is usually not easy to isolate individual basins in the South Florida hydrologic system, the L-8 basin is relatively isolated and therefore somewhat easy to study. Figure 6 shows a site map of the L-8 basin.

The L-8 basin is located within an area approximately 100 km x 100 km in size and near the northern boundary of Palm Beach County, FL. It is bounded by the L-8 canal and the M-canal on the south, the Pratt and Whittney Complex and the Indian Trail Drainage District on the east and the Lake Okeechobee on the west. It includes the Corbett and Dupuis Wildlife areas to the north, part of the Village of Royal Palm Beach (VRPB) to the east, an agricultural area covering citrus to the south, and an agricultural area to the north. Water supply needs of the system include the agricultural demand in the north drawn from the L-8 canal, irrigation withdrawals along the M-canal and water withdrawals from M canal for the City of Palm Beach Utilities Department. eastward water movement along the M-canal is due to the pump station on the canal. The capacity of the L-8 canal is about 14 m^3/s when its water level is about 4.6 m above sea level. The capacity of the M-canal is about 8.5 m^3/s. The M-1 canal drains some of the water in the Village of Royal Palm Beach to C-51 canal.

The L-8 canal is connected to Lake Okeechobee at culvert 10-A at the north end. During the simulation period, excess runoff from L-8 is routed to the lake by gravity during flooding. At the southern end, L-8 is connected to the structure complex S-5A, which is capable of sending water to the south, L-8 canal or the C-51 canal and then the ocean depending on a number of conditions. The model uses a head boundary condition at culvert 10-A and a discharge boundary condition near the structure S5-A. The L-8 canal and all the other canals are fully integrated with the 2-D flow domain except near S5-A where the canal runs by itself. Most of the 2-D domain covered in the model is assumed to have no-flow boundaries. The boundary condition near the agricultural area south of S10A is a constant wall head type boundary condition set to 4.3 m. This boundary condition is used to simulate the effects of present drainage practices approximately.

A number of levees restrict overland flow in the basin. The levee along the L-8 canal is the most prominent and prevents water in the Dupuis and Corbett wildlife areas from entering into the L-8 canal. There are four water control structures and bleeders located in the levee to maintain the water levels in the 5.2-5.8 m range and release the excess to L-8. A second levee, marked as FPL road on the map, runs from north to South between the Dupuis and Corbett areas preventing overland flow between them. A third levee prevents overland flow from the northeastern quarter entering the VRPB except through a culvert structure. The remaining canal sections in the southwestern quarter are assumed to be without levees and are therefore subjected to both stream-aquifer and stream-overland flow interactions.

To demonstrate how water management operations are simulated in the model, the operation of the 720-acre impoundment by the Indian Trail Water Conservation District (ITWCD) is used. The impoundment is used during floods to maintain low water levels in areas draining to M-0 canal. The ITWCD operates pumps sending water into the impoundment when the flood levels at the M-0 canal exceed critical levels. The pump capacity is 31 m^3/s. The outflow from the impoundment passes through three outflow structures with 1.4 m discharge pipes and a 6.4 m invert elevation. The discharges in the pumps can be approximately characterized using a 1-D lookup table in XML similar to the following.

```
<single_control wmID="3" id1="10034" id2="354"
    control="10034" revflow = "yes"
    label="Pump at the impoundment" >
        5.0   0.0
        5.1   6.3
        5.3  12.6
        5.9  25.2
        6.1  31.0
</single_control >
```

The lookup table moves water from a water body with ID=10034 which is the canal segment in M-0 to water body with ID=354 which is a cell in the impoundment starting at a water level of 5.0 m. Pumping is controlled by water level in the water body 10034. When the water level of the control is 5.9 m, the discharge rate is 25.2 m^3/s as shown in the lookup table. The identification tag of the water mover is 3, which is used when assigning an operational logic. The operational logic is managed using a separate section of the model called the management simulation engine (MSE).

The operational logic for the pump directs pump closures when the impoundment water level is above 8.2 m to prevent overtopping, and when the L-8 water level is higher than 4.27 m. The XML tags used to describe the lengthy operational logic for this and other structures in L-8 are not shown in this paper. This example demonstrates that a small basin such as L-8, in South Florida, can become complex because of the operational rules of the pumps and structures.

The total simulation period used is between 1992-1995, while 1994 is used for calibration. The time step selected is 1 day because of data availability. The area is discretized using 1027 cells and 49 canal segments. Figures 6 and 7 show the discretizations. Daily rainfall and potential evapo-transpiration (PET) are provided to the model in a 1.61 km X 1.61 km square mesh. The first seven months of the simulation are used for initialization. Figure 7 shows the water levels and the water velocity vectors one year after the simulation has begun. The figure shows the drainage patterns in the L-8 basin and the confluence of flow into the area where the structures are located. The flow is prevented from moving into Dupuis because of the levee between Dupuis and Corbett. Figure 8 shows water levels at gages marked DUPUIS1 and DUPUIS2 in the northern part of the basin. Figure 9 shows that water levels in the basin very close to the canal are influenced by the canal levels. All the water levels indicate that the model is capable of representing the system reasonably under the natural stresses of the rain and imposed stresses of the L-8 canal. Figure 10 shows the simulated and computed discharges in the L-8 canal for the same period.

The finite volume method and the object oriented code design are responsible for some of the water budget functions of the code. These capabilities are illustrated using water budgets of two water bodies for one day of the simulation. Table 1 shows the water budgets of cell 192 and segment 10008 on December 31, 1992. The tables were created using the list of water mover objects attached to the water bodies. The summation of water volumes in and out of the water bodies are zero at all times when using the finite volume method. The residual volume in the table is mainly due to floating point truncation.

CURRENT AND FUTURE APPLICATIONS

One of the functions of RSM in South Florida is to serve as a regional hydrologic model for many disciplines. Since RSM provides support to accommodate complex site-specific conditions using pseudo cells, SV relations and other features, it is possible to design many complex model applications without making code changes. Scientists familiar with complex local conditions and management rules can develop pseudo cell models without having to calibrate the same set of parameters to different conditions. Development is currently under way to construct pseudo-cell functions for Southwest Florida and and other areas. Current applications of RSM in South Florida include the application in the Water Conservation Area 1 (16292 cells), Loxahatchee river watershed (7247 cells), South West feasibility study area (35000 cells) and the Southern Everglades (52817 cells).

SUMMARY AND CONCLUSIONS

An implicit finite volume method, a high-speed sparse solver, and the object oriented design approach contributed to the development of a fully integrated regional hydrologic model. The model was developed using a number of abstractions such as the water body, water mover, storage-volume (SV) relationship and pseudo cells to accommodate the complex hydrologic features of the system seamlessly into one simple computational algorithm. An object oriented design provided an unlimited capability for the model to expand. The implicit method helped to make it stable.

The model was applied to a small but complex hydrologic basin in South Florida to demonstrate how different hydrologic components with different land use types could be incorporated into one model application. Results show that the model is capable of simulating the water levels and discharges observed in the field. Results also show that the model can provide consistent water budget information for model components.

The error analysis shows that the numerical errors of the model results agree with the error estimates computed using analytical methods developed by Lal (2000). The error tests confirm the accuracy of the solution and the boundary conditions. Analytical estimates of numerical error are extremely useful in designing a suitable model discretizations that can deliver a given accuracy without actually setting up the model.

Table 1: Sample water budgets for two water bodies on Dec 31 st, 1992 in m^3/day

Water Body	Attached water movers	Inflow vol
Cell 192	Overland from Cell 191	0.00
	Overland from 94	0.00
	Groundwater from cell 191	6.56
	Groundwater from cell 193	3178.24
	Groundwater from cell 94	-44.39
	Seepage from seg 10008	-4722.00
	Change in storage	-1268.68
	Mass balance error	2.2×10^{-5}
Segment 10008	Flow from seg 10007	-63347.40
	Flow from seg 10009	51222.50
	Flow in weir	0.00
	Flow in bleeder	0.00
	Seepage from cell 191	6436.07
	Seepage from cell 191	4722.06
	Seepage from cell 96	65.60
	Seepage from cell 94	35.96
	Seepage from ell 95	13.61
	Change in storage	-861.50
	Mass balance error	-8.3×10^{-5}

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APPENDIX II: NOTATION

The following symbols were used in the paper.

A_c	cross sectional area of a canal
A_l	lake area
B	width of a canal segment
$C(H)$	conveyance of overland flow
f	frequency defined as radians per second
$f_{sv}(H)$	stage-volume relationship function converting head to a volume
H	water head
$\mathbf{H}^n$	heads of water bodies at time step n
k	wave number defined as 2π/ wave length.
$\mathbf{M}$	resistance matrix
n_b	Manning coefficient
S	water surface slope
R	hydraulic radius of the canal
R_{rchg}	net contribution to recharge from local hydrology to the regional system
s_c	storage coefficient of the soil
$T(H)$	transmissivity of the aquifer
t	time
U, F, G	conservative variables in the equations of mass balance.
$\mathbf{V}$	volumes of water contained in the water bodies.
W	source and sink terms in the continuity equation
x, y	Cartesian coordinates
z_m	ground elevation of cell m
α	time weighting factor
$\Delta\mathbf{A}$	diagonal matrix of effective areas.

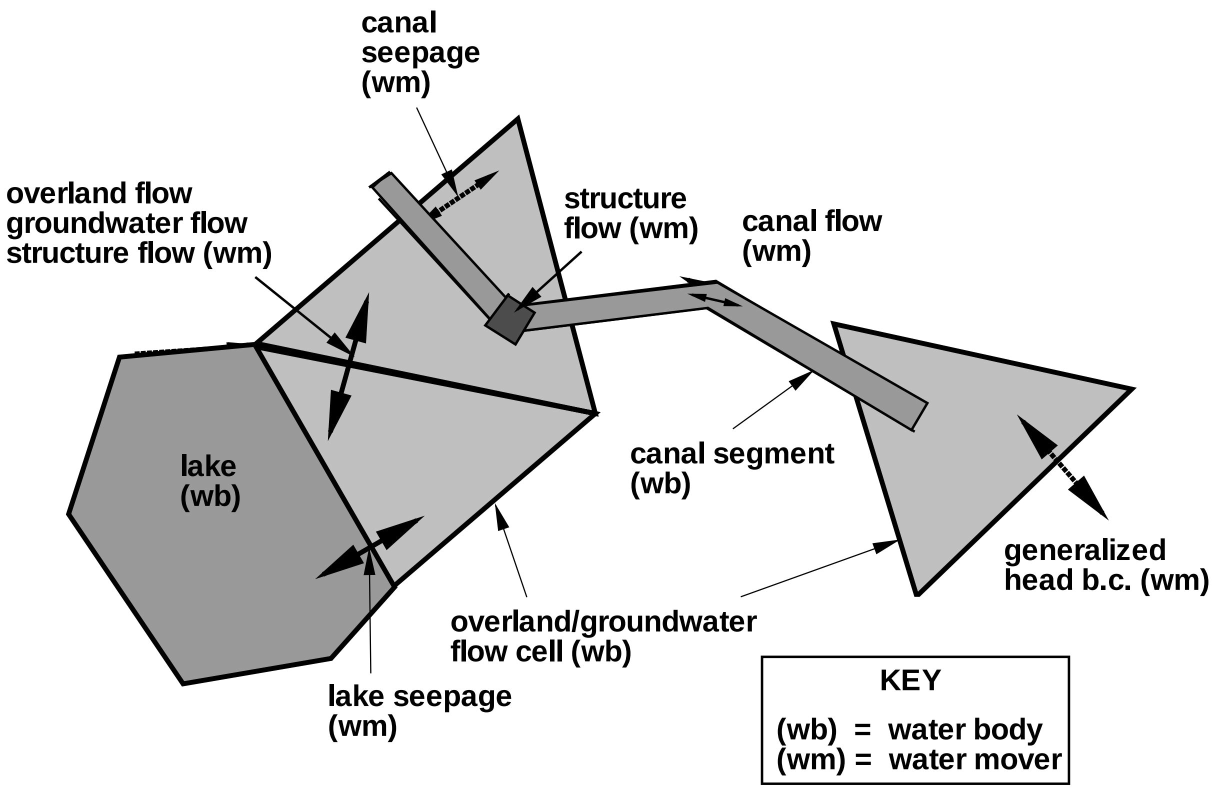

Figure 1: Objects used in the model

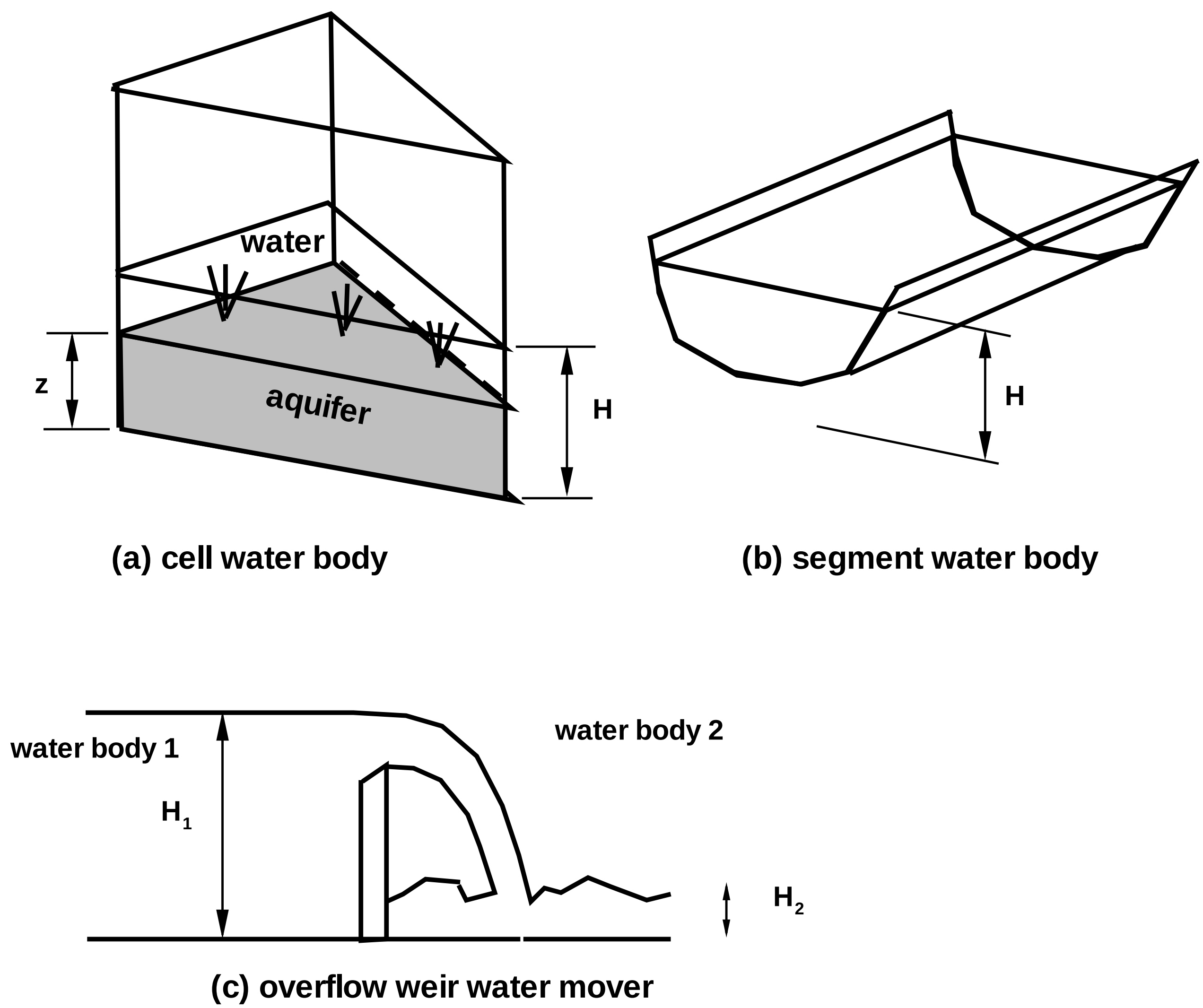

Figure 2: Examples of water bodies and a water mover

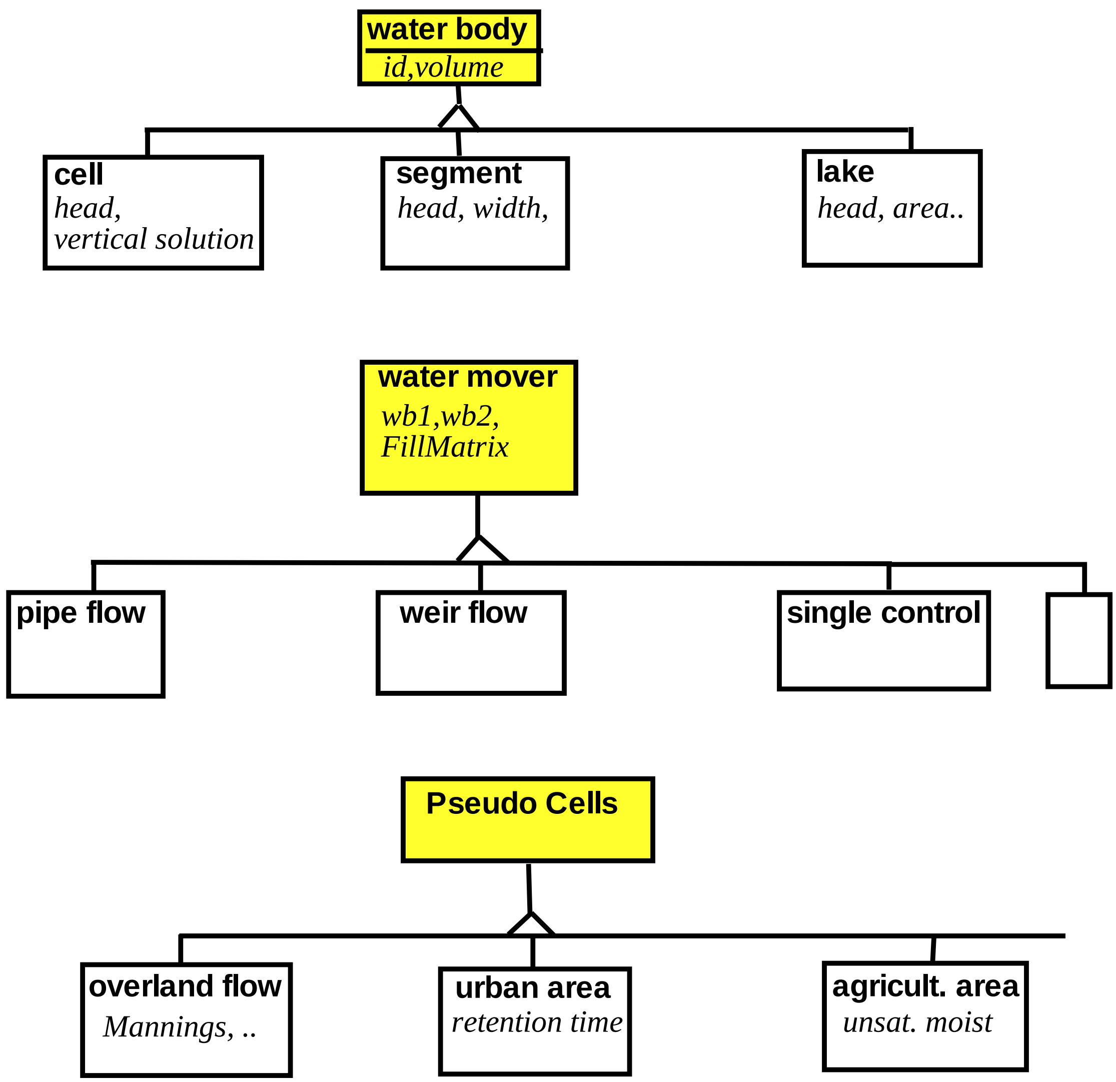

Figure 3: Class diagrams showing the basic building blocks of the model

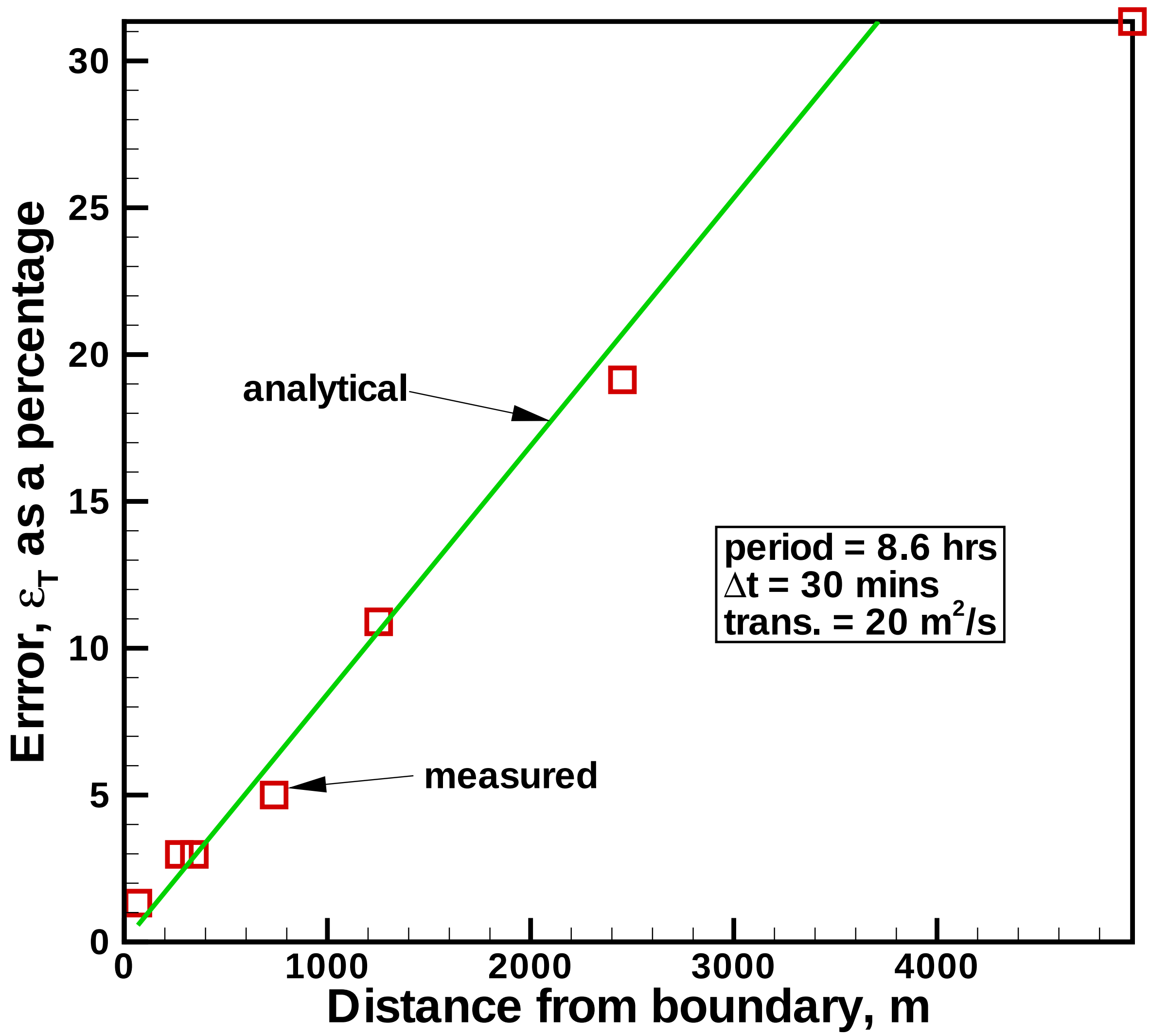

Figure 4: Variation of percentage error with distance

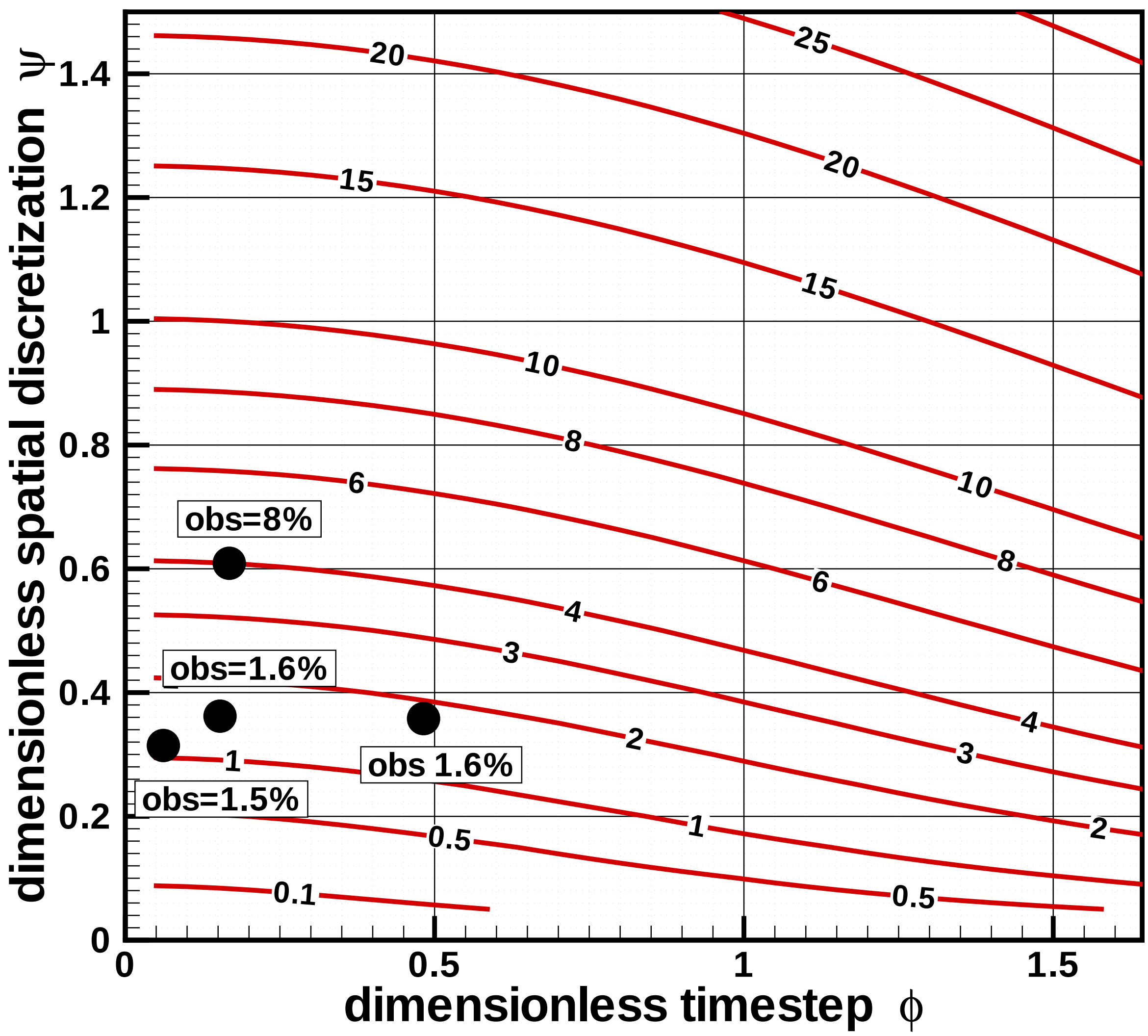

Figure 5: Contours of ε (%) against ϕ and ψ

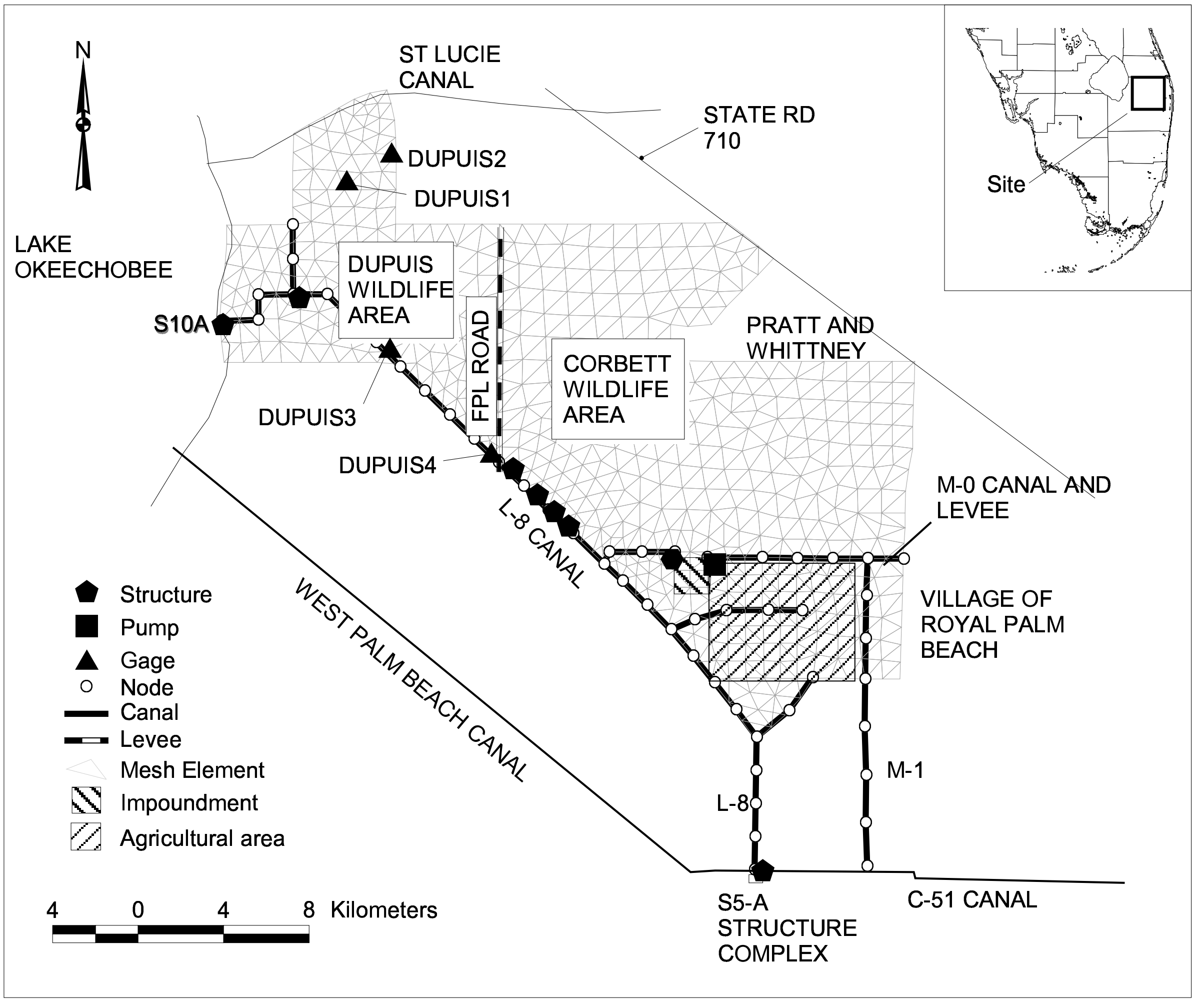

Figure 6: A site map of the L-8 basin

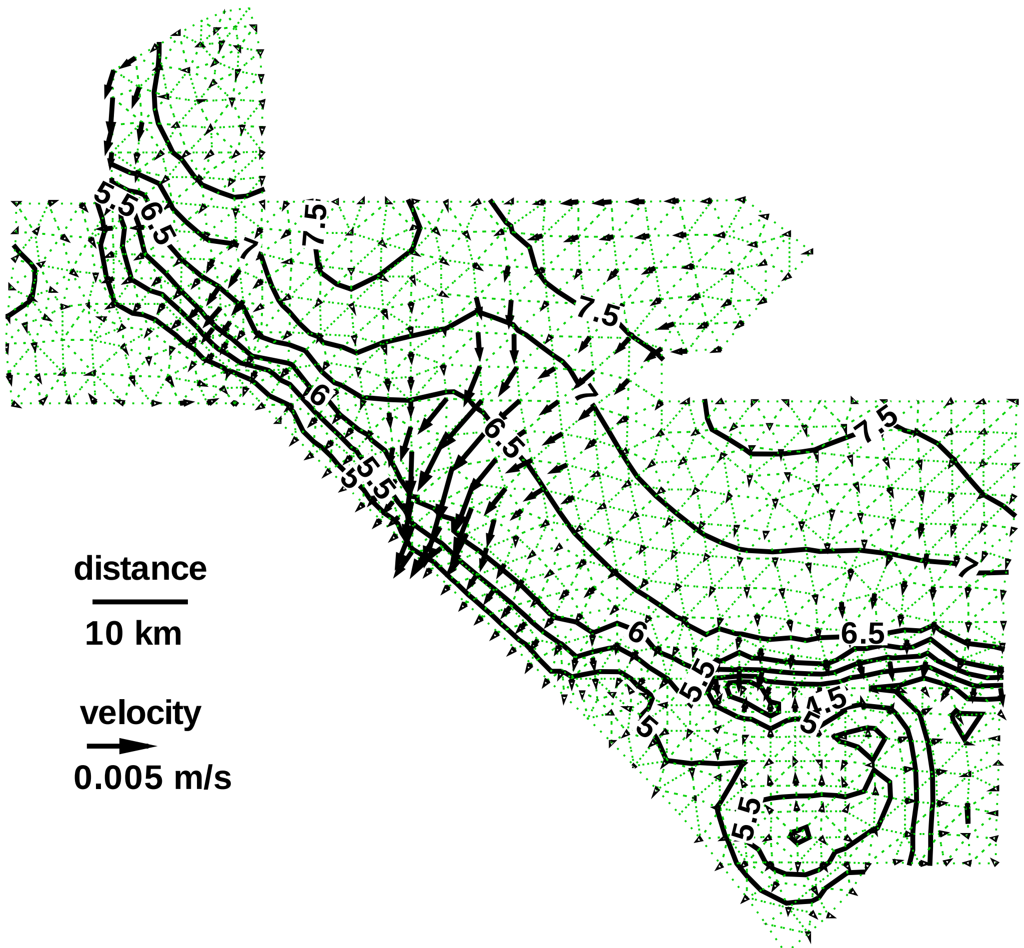

Figure 7: Water levels and flow velocities of L-8 in January, 1993

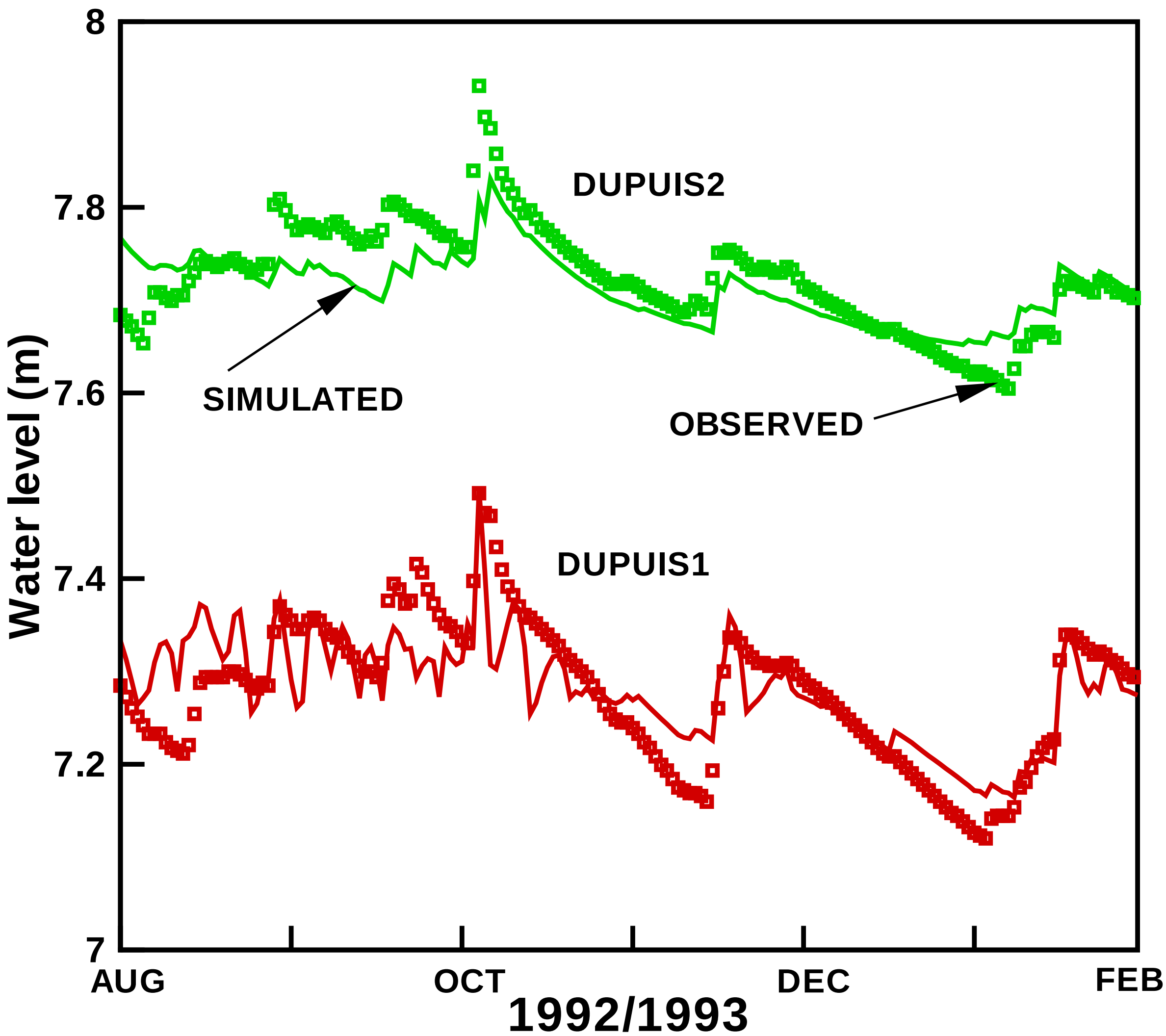

Figure 8: Water levels in Dupuis gages 1 and 2

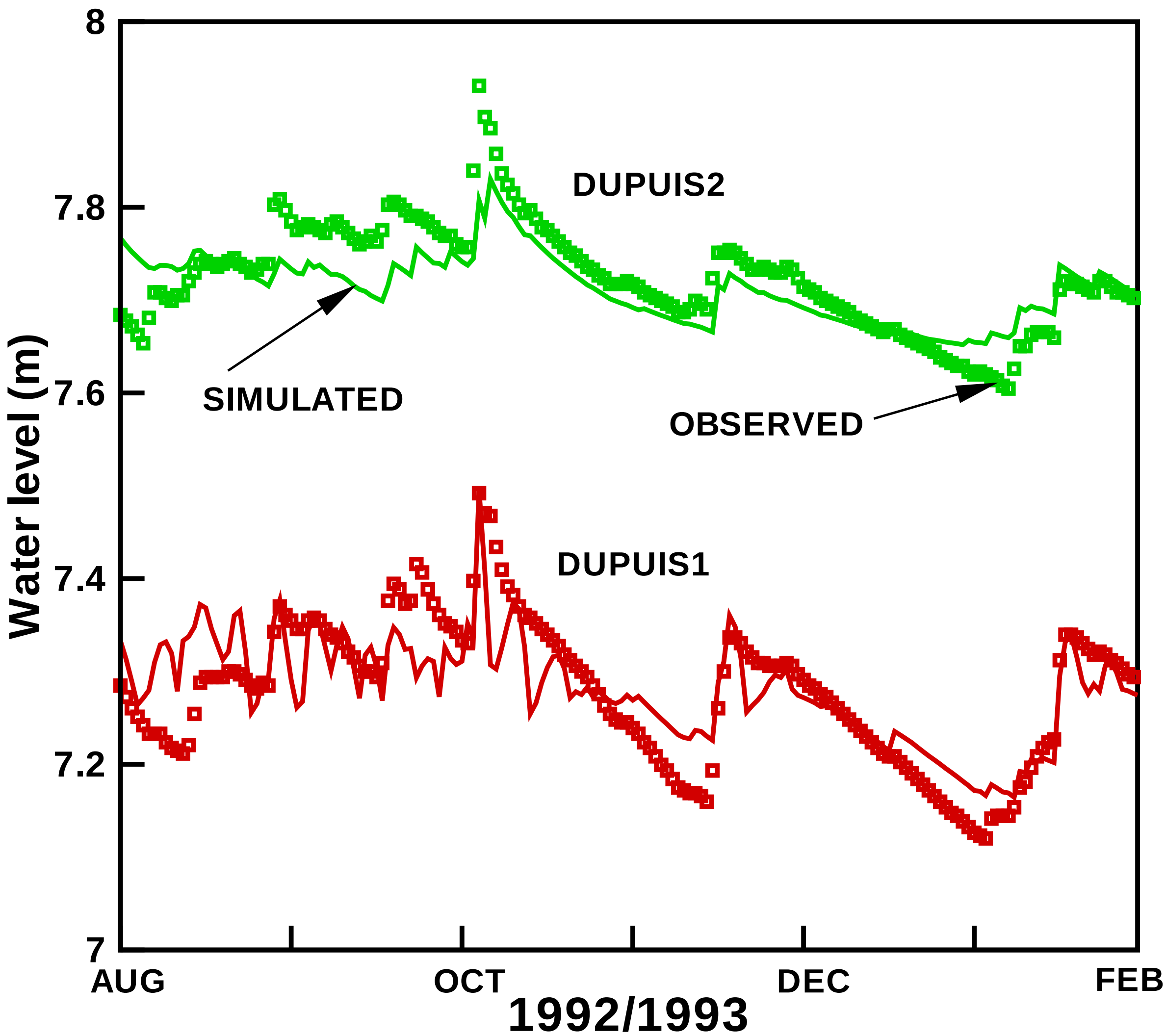

Figure 9: Water levels in Dupuis gages 3 and 4

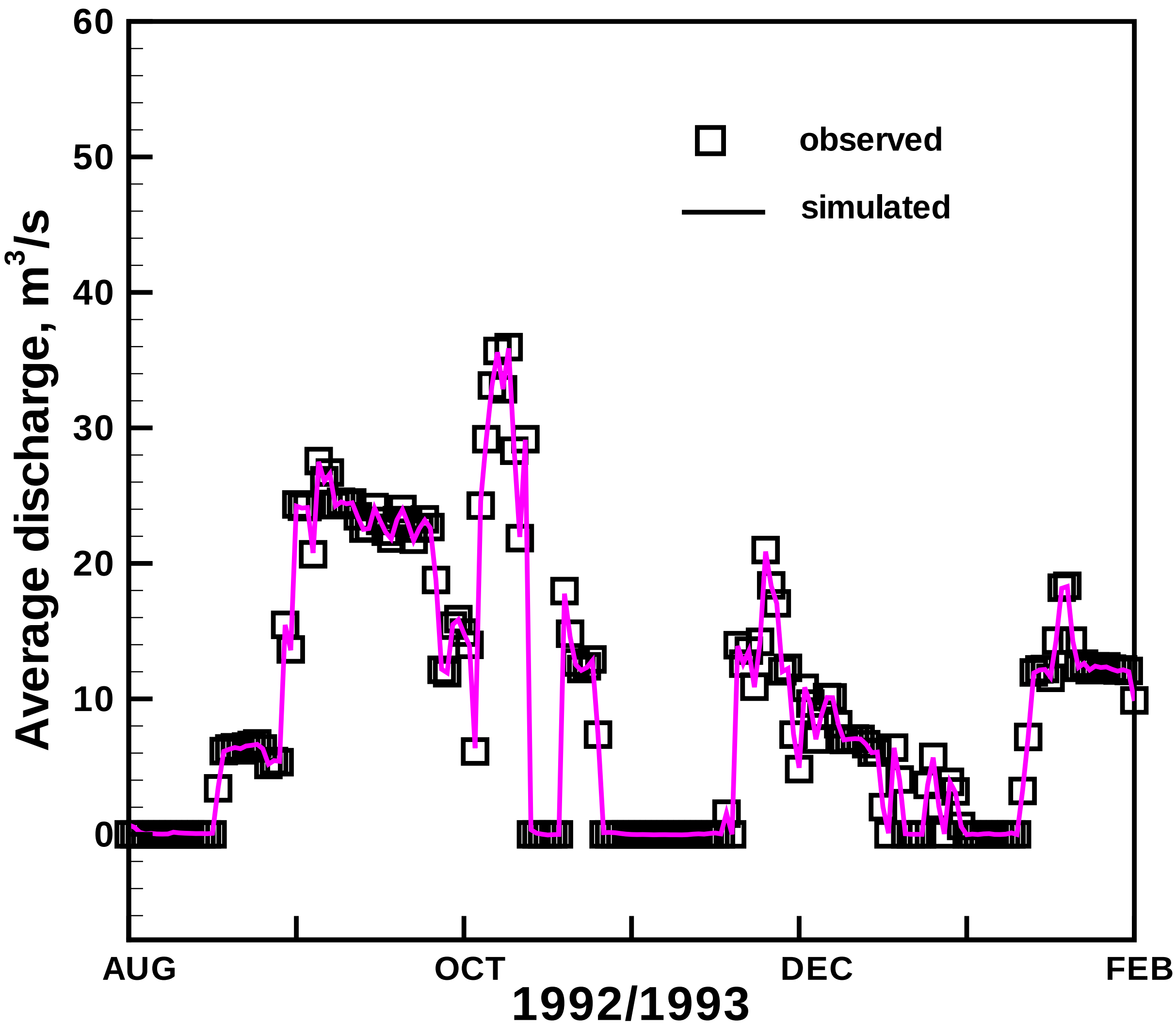

Figure 10: Discharges at S-10A

ε	maximum error per time step as a fraction of amplitude.
ε_T	maximum error over the duration as a fraction of amplitude.
ε_{abs}	the absolute error over the duration
ϕ	dimensionaless spatial discretization
ψ	dimensionaless temporal discretization